FINAL Geotechnical Engineering Report ...

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RAHWAY ARCH PROPERTY

Carteret, New Jersey

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1.0 INTRODUCTION

In accordance with our proposal, Baker has completed the Subsurface Exploration and Geotechnical Evaluation for the proposed Rahway Arch project, located in Carteret, New Jersey (Figure A-1).

The purpose of this study was to determine general subsurface conditions at the project site and to evaluate those conditions with respect to geotechnical engineering considerations for the proposed construction. The specific scope of our services on this project consisted of the following.

- Drilling and Fieldwork, consisting of 31 auger borings and ten (10) cone penetrometer test with pore pressure measurements (CPTu) borings.
- Laboratory testing consisting of water content, Atterberg limits, and grain-size distribution, USCS textural classification, specific gravity, pH, organic content, permeability, consolidation analyses, unconsolidated-undrained triaxial tests, and consolidated-undrained (CIU) triaxial tests.
- A review and description of the field and laboratory test procedures conducted and their results;
- A review of area and site geologic conditions, including geological hazards at the site, such as soft soils, swelling soils, sensitive soils, liquefaction, etc.;
- A review of subsurface conditions encountered with available physical properties;
- Potential excavation difficulties;
- Results of slope stability and settlement analyses;
- Recommendations for constructing the embankment/cap;
- Recommendations for a geotechnical monitoring program;
- Recommendations for shallow foundations (Net allowable bearing pressure and applied safety factor, recommended bearing depth, resistance to sliding, resistance to uplift, estimated settlement and modulus of subgrade reaction);
- Recommendations for deep foundations, if necessary;
- Subsurface drainage and potential difficulties with groundwater;
- Seismic site classification and recommendations;
- Site preparation, subgrade preparation, and construction and testing compacted fills;
- Other geotechnical concerns that may affect the planned construction; and
- A review and comment on the final remedial plan design for consistency with the geotechnical recommendations.

2.0 SITE AND PROJECT DESCRIPTION

In this section, details of the areas explored are described based on information available at the time of this report. A conceptual design site plan, indicating the location of the buildings, site boundaries, and associated parking; a topographic survey drawing; and past geotechnical and environmental reports were provided for this report by Soil Safe, Inc. (Soil Safe).

2.1. SITE DESCRIPTION

The Rahway Arch site is located on the banks of the Rahway River in the Borough of Carteret and was used from the 1930s through the 1970s for disposal of a mixture of alum sludge and yellow prussiate of soda (YPS) sludge from the American Cyanamide Warner Plant in Linden, New Jersey. The overall 124.7 acre site contains six impoundments, encompassing approximately 85 acres, located on the Rahway River. The impoundments were constructed above existing grade with wooden and earthen dikes. They contain approximately 2,000,000 tons of the cyanide containing alum-YPS sludge. The thickness of the sludge ranges from 5 to 20 feet.

The majority of the site is lightly vegetated with cattails and similar vegetation, with a relatively flat topography. The site is bounded to the northwest, north, northeast, and east by the Rahway River. To the west is marshland and to the south and southeast are tank farms. Site elevations vary between +13 ft above mean sea level (amsl) on the north portion of the property to sea level along the wetlands bordering the site.

2.2. PROJECT DESCRIPTION

The owner has entered into an agreement with Soil Safe to be the reclamation contractor and construct a cap over the impoundments. Soil Safe will be constructing a Class B soil recycling facility on Impoundment 2 to manufacture the engineered fill required for the cap. Impoundment 2 will be capped at the end of the project; after all other impoundments have been capped. The thickness of the soil cap will be determined, in part, from the study contemplated herein. In addition, the Soil Safe process will require the use of large temporary soil stockpiles for both pre-process and post-process soil material. For the purpose of this investigation, Baker will assume soil placement thicknesses in the range of five (5) to thirty (30) feet for the cap and possibly an additional twenty-five (25) feet for the soil stockpiles. Based upon historic data, the subsurface profile consists of surface fill, alum-YPS sludge, peat, organic silts and clays, and alluvial sand and glacial deposits (clay, silt, sand, and gravel) overlying red-brown siltstone and shale.

In order to construct this cap, the shear strength and compressibility of the underlying materials must be determined. This includes the time-dependent stress behavior of the underlying materials in order to formulate a time sequenced soil placement plan to safely construct the soil cap. Specific objectives include:

- characterizing the subsurface conditions in the impoundments, berms and the adjacent wetlands outside the berms;
- determining the stability of the berms for the existing conditions and placement of an engineered fill cap over the impoundments;
- evaluating the geotechnical conditions in the proposed areas for the Class B facility, the scale and the steel bridge that allows access of the site;
- evaluating stability of the impoundments to support the engineered fill cap;
- performing slope stability analyses to ensure that adequate factors of safety can be maintained in the final cap design;
- estimating the settlement that will occur in the impoundments;
- determination of the effective vertical overburden pressure by area and related time rate of consolidation allowable to safely place the soil cap and temporary stockpiles;
- develop construction options for handling excessive settlement and slope stability issues; and
- develop a geotechnical monitoring program to be implemented during construction (along with contingency options for excessive settlement and slope stability issues).

3.0 GEOLOGIC SETTING

3.1. SITE GEOLOGY

Based upon the Surficial Geology Map of the Perth Amboy and Arthur Kill Quadrangles (Stanford, 1999) and the Geologic Map of New Jersey (Drake, etal, 1996), the proposed development will be sited primarily over a relatively thin layer of surficial unconsolidated deposits and bedrock below that. Surficial unconsolidated materials in the Perth Amboy and Arthur Kill quadrangles consist of glacial, stream, wetland, and weathered bedrock sediment. The glacial sediment is not found at the Rahway site. The stream sediment, as much as 40 feet thick, includes sand, gravel, and silt deposited in floodplains, stream terraces, and former river plains. The wetland sediment includes peat and organic silt and clay deposited in freshwater swamps and saltwater marshes and estuaries. It is as much as 100 feet thick. The weathered bedrock consists of silty clay and shale fragments formed by chemical and mechanical decomposition of shale bedrock of Triassic and Jurassic Age. It is generally less than 10 feet thick. In the area of the Rahway Arch property, the primary surficial unconsolidated deposits are pre-glacial weathered bedrock and postglacial artificial fill and estuarine and salt-marsh deposits. The underlying bedrock is comprised of Passaic and Lockatong Formation sedimentary rock. Specific details of the units found at the site are included below:

Artificial Fill overlying Estuarine and Salt-Marsh Deposits (af/Qm). Artificial fill is made up of excavated sand, silt, clay, gravel, rock, and till, and man-made materials (bricks, cinders, ash, slag, glass, construction materials and minor amounts of trash). Color is variable, but generally gray to black. The unit is as much as 50 feet thick, but is generally less than 20 feet thick. At the Rahway site, the artificial fill is primarily YPS-alum sludge.

Estuarine and Salt-Marsh Deposits (Qm). Brown to dark gray, peat and organic clay and silt, with minor sand and shells. Locally, at the base of this unit, alluvial sand and gravel, deposited before marine inundation, may be present. The thickness of the unit may be as thick as 100 feet.

Weathered Shale (Qsw). Poorly-sorted, nonstratified to weakly stratified, reddish-brown to yellowish-red silty clay to clayey silt with some to many angular to subangular chips of red (and minor gray) shale. Derived from mechanical and chemical decomposition of shale of the Passaic Formation of Triassic and Jurassic Age. The unit is generally less than 10 feet thick.

Passaic Formation (JTrp). This Lower Jurassic and Upper Triassic Age formation, previously known as the Brunswick Formation, is comprised of predominantly red beds consisting of argillaceous siltstone; silty mudstone; argillaceous, very fine grained sandstone; and shale; mostly red-brown to brown-purple and gray-red. The red beds occur typically in 10 to 23 feet thick, cyclic playa-lake-mudflat sequences and fining-upward fluvial sequences. Lamination is commonly indistinct due to borrowing, dessication, and paleosol formation. Where layering is preserved, most bedforms are wavy parallel lamination and trough and climbing-ripple cross lamination. Calcite- or dolomite-filled vugs and flattened cavities, mostly 0.02 to 0.08 inch across, occur mostly in the lower half. Sand-filled burrows, 0.08 to 0.2 inch in diameter, are prevalent in the upper two-thirds of the unit. Dessication cracks, intraformational breccias, and curled silt laminae are abundant in the lower half. Lake cycles, mostly 7 to 16 feet thick, have a basal, greenish-gray, argillaceous siltstone; a medial, dark-gray to black, pyritic, carbonaceous, fossiliferous, and, in places, calcerous lake-bottom fissile mudstone or siltstone; and an upper thick-bedded, gray to reddish and purplish-gray argillaceous siltstone with dessication cracks, intraformational breccias, burrows, and mineralized vugs. The thickness of the formation is about 11,500 feet.

Lockatong Formation (Trl). This Upper Triassic formation is comprised of predominantly cyclic lacustrine sequences of silty, dolomitic or analcime-bearing argillite; laminated mudstone; silty to calcareous, argillaceous very fine-grained sandstone and pyritic siltstone; and minor silty limestone, mostly light- to dark-gray, greenish gray, and black. Grayish-red, grayish-purple, and dark brown-red sequences occur in some places, especially in the upper half. Two types of cycles are recognized: freshwater-lake (detrital) and alkaline-lake (chemical) cycles. Freshwater-lake cycles average 17 feet thick and consist of basal, transgressive, fluvial to lake-margin deposits that are argillaceous, very fine-grained sandstone to coarse siltstone with indistinct lamination, planar or cross lamination, or are disrupted by convolute bedding, dessication cracks, root casts, soil-ped casts, and tubes. Medial lake-bottom deposits are laminated siltstones, silty mudstones, or silty limestones that are dark gray to black with calcite laminae and grains and lenses, or streaks of pyrite; fossils are common, including fish scales and articulated fish, conchostracens, plants, spores, and pollen. Upper regressive lake margin, playa lake, and mudflat deposits are light- to dark-gray silty mudstone to argillitic siltstone or very fine-grained sandstone, mostly thick-bedded to massive, with dessication cracks, intraformational breccias, faint wavy laminations, burrows, euhedral pyrite grains, and dolomite or calcite specks. Alkaline-lake cycles are similar to freshwater-lake cycles, but are thinner, averaging 10 feet, have fewer fossils (mainly conchostracens), and commonly have red beds, extensive dessication features, and abundant analcime and dolomite specks in the upper parts of cycles. The thickness of the formation near Byram is about 3,500 feet. The formation thins to the southeast and northeast, with the thickness less than 2,300 feet near Princeton.

The Passaic Formation underlies the majority of the site, whereas the Lockatong Formation underlies Impound 1 and possibly portions of Impound 2. There do not appear to be any major faults close to the site, although any structural features of the basement rock underlying the site are hidden by the overlying unconsolidated deposits.

3.2. SITE HYDROGEOLOGY

The hydrogeology of the site is dominated primarily by the Rahway River and sea level tidal fluctuations, with shallow groundwater flow generally toward the river. Deeper groundwater within the underlying Passaic Formation bedrock flows seaward. The Passaic Formation is a major source of groundwater to the west of the site, with flow occurring primarily in fractured shale. Separating these two aquifers is a continuous layer of red-brown clay. The clay layer identified beneath the shallow unconsolidated material functions as a confining unit for the underlying Passaic Formation (Hydrosystems, 1989). As such, the clay layer will restrict the vertical flow of water between the shallow and bedrock aquifers. Within the impounds, a groundwater mound composed of freshwater roughly five to ten feet above the groundwater elevation of the surrounding areas has developed, creating a horizontal and vertical flow field within the impounds flowing radially outward to the adjacent surface waters of the Rahway River, Cross Creek, and Deep Creek. The natural groundwater in the area is generally brackish, therefore the impound groundwater is less dense and therefore floats above the brackish groundwater table (Hydrosystems, 1989).

Eight paired monitoring well clusters were installed at this site to monitor the shallow unconsolidated and bedrock aquifers. The shallow monitoring wells were screened from depths of 10 to 20 feet in the shallow fill material and tidal marsh deposits. The deep wells were screened in the upper weathered portion of the Passaic Formation at depths ranging from 40 to 60 feet below ground surface (bgs). The water table is encountered approximately 2 feet bgs in shallow monitoring wells. Water table mounding occurs in the shallow aquifer beneath the impoundments, where ground-water elevation was measured as approximately 10 feet above mean sea level (Hydrosystems, 1989).

The fine-grained Passaic Formation typically has low primary porosity. Where coarser-grained rock is present, it is tightly cemented and has a high clay mineral content. Ground water flow occurs primarily in bedding plane fractures or in secondary fractures (joint sets) formed by stresses related to faulting following the deposition and lithification of the beds (USGS, 1968). Regional flow in the Passaic Formation occurs vertically and laterally toward the northeast, with ultimate discharge to surface water bodies which, in the vicinity of the Carteret Impoundments, include the lower Rahway River, Arthur Kilt, and, eventually, the Atlantic Ocean (Blasland, etal., 1995). Disko (1982) completed permeability tests on subsurface samples. Using data presented in Disko (1982), Blasland, etal. (1995) estimated a mean coefficient of permeability (k) for the sludge fill and tidal marsh units. The mean k value derived from these data was 1.10 ft/day.

3.3. SITE SEISMIC HAZARDS

It is unknown whether a seismic hazard assessment has been performed for this site or for any sites nearby. Historical seismicity within the New York/New Jersey area indicates that over the past 300 years, there have been a number of significant seismological events. Earthquakes with a maximum modified Mercalli scale of VII (roughly between 5.5 and 6.0 on the Richter scale) occurred in the New York City area in 1737, 1783, and 1884 (Dombroski, 2005). A number of smaller events have also occurred in the area over the last 300 years. The time spans between events indicate a 100 year return period. Based upon review of a geology map of New Jersey, there are no known faults on or near the site. However, there are many faults in New Jersey including the Ramapo Fault, separating the Piedmont and Highlands Physiographic Provinces, to the northeast of the site.

Utilizing current data developed from earthquake measurements in the region, the peak horizontal ground acceleration with a 7% probability of exceedance in any 75-year period ranges between 88 gals and 98 gals or 0.09g to 0.10g (AASHTO, 2008). The spectral acceleration at 0.2 second period with a 7% probability of exceedance in any 75-year period ranges between 157 gals and 177 gals or 0.16g to 0.18g and the spectral acceleration at 1.0 second period with a 7% probability of exceedance in any 75-year period ranges between 29 gals and 39 gals or 0.03g to 0.04g. Peak acceleration is the acceleration experienced by a particle on the ground. Spectral acceleration is approximately what is experienced by a building, as modeled by a particle on a mass-less vertical rod having the same natural period of vibration as the building.

Most methods for determining seismic soil response are based upon the assumption that upward propagation of horizontally polarized shear waves from the underlying rock formation governs the response of the soil deposit. Two independent design response spectra are typically developed, one to define the horizontal component of ground motion, and the second to define the vertical component. The vertical component of ground motion usually contains much higher frequency content than the horizontal component; therefore the spectral shape is different than that of the horizontal component. The peak ground acceleration (PGA) associated with the vertical component will also be different than the PGA of the horizontal component. Both values of PGA are dependent on the distance from the source.

The type of soil affects the response to dynamic loading. The most significant factors include grain size distribution, clay fraction, and degree of saturation. For sensitive cohesive soils, such as those that exist at the site, liquefaction and seismic response may be important. Embankment slope materials that are vulnerable to earthquake loadings include very steep, weak, fractured, and brittle rocks or unsaturated loess; loose saturated sand; sensitive cohesive soils with natural moisture exceeding the liquid limit; and dry cohesionless material on slopes at the angle of repose.

4.0 HISTORICAL DATA

4.1. PREVIOUS INVESTIGATIONS

There have been a number of geotechnical and environmental-related studies conducted at the Rahway Arch property. M. Disko Associates (Disko, 1981a) conducted the earliest study on the impounded sludge in June 1981, drilling twelve (12) borings inside the impounds (one at the edge and one at the center of each impound) to the bottom of the sludge layer. Twelve (12) in-situ density tests on the sludge were made using the Sand Cone Method (ASTM D1556). The resulting sludge densities ranged from 36.4 pcf to 81.9 pcf with an average in-situ density of 54.7 pcf. The moisture content was not tested. Falling head permeability tests were also run on remolded samples of sludge, with permeability ranging from $6.57(10)^{-6}$ cm/sec to $1.19(10)^{-4}$ cm/sec, averaging $5.33(10)^{-5}$. Field conditions within the impounds were also noted. Pond #1 was the only pond that was covered by vegetation. The sludge also showed signs of stratification and coloring within all of the impounds, being most pronounced within Impounds 2, 3, and 4.

In September 1981, Disko (1981b) again conducted a sludge investigation, drilling twelve (12) borings and conducting laboratory permeability tests. The intent was to evaluate permeability closer to the base of the sludge layer. The borings were again drilled at the edge and center of each impound to depths ranging from 10 feet at the edge of Impound 1 to 29 feet at the center of Impound 6. Sludge depths ranged from 5 feet at the edge of Impound 2 to 20 feet at the edge of Impound 5. The borings were terminated in the organic silt layer underlying a layer of peat. Falling head permeability tests were run on remolded samples of sludge 1 to 2 feet above the bottom of the sludge layer and on remolded samples of the underlying silt collected 3 to 13 feet below the sludge/soil interface. Permeability values of the bottom of the sludge ranged from $5.90(10)^{-6}$ cm/sec to $8.00(10)^{-5}$ cm/sec, averaging $2.46(10)^{-5}$ cm/sec. The permeability values of the silt layer ranged from $2.20(10)^{-7}$ cm/sec to $6.34(10)^{-6}$ cm/sec, averaging $2.46(10)^{-6}$ cm/sec.

A third investigation was conducted by Disko (1982) in January 1982 to evaluate the condition and soil material of the earth berms surrounding the impounds and to evaluate what type of material they were constructed upon. The borings were drilled through the berms and underlying silt material and to the underlying rock formation (for three of the borings). Eleven (11) borings were drilled at the site, B-1 to B-10 and B-6A. Three (3) borings were drilled to the top of rock at depths ranging from 29 feet to 38.5 feet. Sludge was found below the earth berms in eight (8) of the eleven (11) borings. The berm was found to be lying directly on peat in three (3) borings. Falling head permeability tests were also run on remolded samples of sludge, berm material, peat, organic silt, sand and gravel, and shale fragments. The permeability of the berm material ranged from $3.00(10)^{-2}$ cm/sec to $5.00(10)^{-6}$ cm/sec. The permeability of the materials were similar to past test results.

In 1997, Blasland, Bouck, and Lee (BBL, 1997) conducted a geotechnical exploration at the site, performing SPT drilling, Shelby tube sampling, field vane shear testing, and laboratory analysis. Seven (7) soil borings (SB1 to SB4, SB6 to SB8) were drilled within the Impounds 2, 3, 4, and 5 to collect Shelby tube samples and to determine the depth of the sludge. A total of 56 vane shear tests were also performed within all of the sludge impoundments conducted using a Geonor H-60 at 13 locations and at depth of 1, 2, 5 and 9 feet below the ground surface. At locations further into the impounds, the tested undrained shear strength ranged from 115 psf to 940 psf with an average strength of 482 psf. At test locations near the berms, the undrained shear strength varied from 668 psf to 1337 psf with an average of 981 psf. The results from locations in Impound 6 have the lowest undrained shear strength. Remolded vane shear tests were also performed with results ranging from 0 to 574 psf, averaging 47.6 psf.

The “undisturbed” Shelby tube sludge samples were tested in a geotechnical laboratory. A total of 13 samples from Borings SB1 to SB4 and SB6 to SB8 were tested for index properties, moist and dry density, specific gravity, shear strength testing, and consolidation testing. The sludge samples were characterized as elastic silts and silts with moisture contents ranging from 69.3% to 128.9% with an average of 93.8%. The tested moist density ranged from 60.3 to 102.7 pcf with an average moisture density of 92.5 pcf. Specific gravity values ranged from 2.89 to 3.27 with an average value of 3.11. The initial void ratio ranged from 2.295 to 3.948 with an average of 3.073.

A total of seven (7) undisturbed sludge samples were laboratory tested for unconsolidated undrained (UU) shear strength in 1997 by BBL (1997). The applied confining pressure in the triaxial chamber for the UU testing is approximately equivalent to the total overburden pressure at the sample depth. The tested undrained shear strength ranged from 215 to 1085 psf depending on the sample location. Three (3) undisturbed sludge samples were tested using the direct shear test method under drained conditions in 1997 by BBL. The samples were tested under 1, 3 and 10 ksf vertical pressures. The tested peak drained shear strength was 35º internal friction angle and 300 psf cohesion. If the cohesion is neglected, the equivalent internal friction angle under each vertical loading varies from 36º to 48º. Four (4) consolidation tests were also performed on undisturbed samples of the on-site sludge. Compression and recompression indices ranged from 0.767 to 1.152, averaging 0.965 and 0.02 to 0.022, averaging 0.021, respectively. Estimated preconsolidation stresses ranged from 0.9 to 3.1 tons per square foot (tsf) with the highest stress reported from a sample collected within two (2) feet of ground surface, indicating that drying of the material over time has potentially created an overconsolidated material.

In 2006, Edwards and Kelcey (2006) conducted a geotechnical investigation for the Tremley Point

Connector Road, Interchange 12 Improvements Project, for the New Jersey Turnpike Authority (NJTA). The exploration program consisted of drilling and sampling 32 SPT soil borings, installing two (2) observation wells, and performing laboratory testing on select soil and rock samples. The investigation covered wetland areas and three alternate alignments adjacent to the Rahway Property (then owned by Cytec) as well as a portion of the Rahway Property within Impound 3. All borings were drilled to the top of bedrock, which was encountered between 20 and 50 feet below ground surface. Eight (8) unconfined compression tests and ten (10) consolidation tests were performed on undisturbed samples of organic clayey silt to peat with varying amounts of sand. Undrained shear strength values ranged from 60 psf to 290 psf, averaging 181 psf. Compression and recompression indices ranged from 0.062 to 5.127, averaging 1.285 and 0.006 to 0.959, averaging 0.159, respectively. Estimated preconsolidation stresses ranged from 0.2 to 4.7 tons per square foot (tsf).

5.0 FIELD AND LABORATORY WORK

5.1. FIELD EXPLORATION

The field exploration consisted of drilling thirty-one (31) standard penetration test (SPT) borings and ten (10) CPTu borings. Warren George, Inc. (Warren George) completed the borings using a swamp buggy-mounted drill rig and a truck-mounted drill rig from July 10th to August 3rd, 2012 using flush joint casing with mud rotary drilling (casing advancer system) methods to drill the borings. “Tiger” mud and powdered sodium-bentonite were circulated within the hole to remove cuttings. Rock coring was not conducted. All CPT borings were performed from the swamp buggy using a drill rig to advance the CPT rods and probe. Baker personnel logged the borings.

The boring locations were staked by Eaststar Environmental (Eaststar) personnel. Final surveyed locations and elevations at the as-drilled boring locations were provided by Kernan Consulting Engineers (Kernan). Current and historical boring locations are shown in Figure A-2, located in Appendix A. All SPT and CPT borings were drilled and sampled to the depths shown in Table 5-1.

Table 5-1. Boring Locations and Depths

Boring	Depth (ft)	Location	Boring	Depth (ft)	Location
BD-01R	41.5	Impound 4 Berm	IS-02	27.0	Impound 2
BD-02	32.0	Impound 3 Berm	W-01	41.0	Wetland adjacent to Impound 4
BD-03	47.0	Impound 4 Berm	W-02	26.5	Wetland adjacent to Impound 4
BD-04R	39.0	Impound 2 Berm	W-03	30.0	Wetland adjacent to Impound 3
BD-05	37.0	Impound 1 Berm	W-04	32.0	Wetland adjacent to Impound 1
BS-01	32.0	Impound 6 Berm	W-05	42.0	Wetland adjacent to Impound 5
BS-02	32.0	Impound 3 Berm	W-07R	27.0	Wetland adjacent to Impound 2
BS-03	26.0	Impound 4 Berm	W-08	32.0	Wetland adjacent to Impound 3
BS-04	27.0	Impound 4 Berm	CP-01	28.9	Impound 4
BS-05	27.0	Impound 1 Berm	CP-02	29.5	Impound 4
BS-06	24.0	Impound 1 Berm	CP-03	19.8	Wetland adjacent to Impound 2
BS-07R	27.0	Impound 2 Berm	CP-04	26.6	Impound 6
BS-08	25.0	Impound 3 Berm	CP-05	39.5	Impound 5
BS-09	26.0	Access Bridge	CP-06A	26.9	Impound 2
ID-01	49.0	Impound 4	CP-08	24.1	Impound 6

Table 5-1. Boring Locations and Depths					
Boring	**Depth (ft)**	**Location**	**Boring**	**Depth (ft)**	**Location**
ID-02	47.0	Impound 5	CP-08A	20.8	Impound 6
ID-03R	34.5	Impound 6	CP-09D	18.9	Impound 3
ID-04	36.0	Impound 3	CP-10	21.0	Wetland adjacent to Impound 6
ID-05	47.0	Impound 2	CP-11A	21.2	Impound 2
ID-06	37.0	Impound 1	CP-12	20.8	Impound 1
IS-01	27.0	Impound 2/5			

5.1.1. Standard Penetration Testing

The field exploration consisted of SPT soil samples obtained at a continuous 2-feet interval for the top 10-feet of drilling, and at 5-feet intervals below 10-feet, except for within the impounds, where continuous sampling was conducted through the entire depth of the sludge. In general, the SPT consisted of advancing a sampling spoon (2-inch outside diameter) 2-feet by driving it with a 140-pound hammer falling 30-inches. Typically, an 18 inch spoon is driven, however, a 24-inch sampling spoon was used for this project. The values reported on the boring logs are the blows required to advance four successive increments. The first 6-inch increment is considered as seating. The sum of the number of blows for the second and third increments is the "N" value. The fourth value is not used, but is recorded on the logs. The soils were classified in general accordance with the Unified Soil Classification System.

5.1.2. Cone Penetrometer Testing with Pore Pressure Measurements

The CPTu boring program took place on July 10th, 12th, 13th and 16th, 2012. A total of eighteen (18) soundings were completed at eleven (11) different sounding locations. The CPT program was performed to evaluate in situ geotechnical criteria relative to the soils. In addition to the CPT soundings, shear wave velocity tests were performed at eight (8) of the locations with testing at five-foot depth intervals and dissipation tests were performed at nine (9) locations at various depths. The cone penetrometer tests were carried out using an integrated electronic piezocone. The piezocone used was a compression model cone penetrometer with a 15 cm^2 tip and a 225 cm^2 friction sleeve. The cone is designed with an equal end area friction sleeve and a tip end area ratio of 0.80. The piezocone dimensions and the operating procedure were in accordance with ASTM standard D-5778-07.

Pore pressure filter elements, made of porous plastic, were saturated under a vacuum using silicone fluid as the saturating medium. The pore pressure element was six millimeters thick and was located immediately behind the tip (the u_2 location) for all soundings. The cone was advanced using a skid drill rig, mounted on a Kori rig operated by Warren George. The following data were recorded onto magnetic media every five centimeters (approximately every two inches) as the cone was advanced into the ground: Tip Resistance (qc), Sleeve Friction (fs), and Dynamic Pore Pressure (u).

During seismic testing, the seismic signals were recorded using a geophone mounted in the cone and an up-hole digital oscilloscope. A sledge hammer, struck against a steel wedge was used as the seismic source. While stopped, pore water pressures were automatically recorded at five-second intervals and the readings stored in a dissipation file for estimation of C_h, the coefficient of consolidation that can in turn be used to calculate K_h, the horizontal hydraulic conductivity.

5.2. LABORATORY ANALYSES

The laboratory testing consisted of performing classification and index testing; including natural moisture content, grain-size distribution, Atterberg limits, and hydrometer analysis; specific gravity; pH; organic content; consolidation analyses; unconsolidated-undrained triaxial tests; and consolidated-undrained (CIU) triaxial tests as shown in Table 5-2 below.

Table 5-2. Laboratory Testing

Laboratory Analysis	ASTM Standard	Purpose
Natural Moisture Content	D2216	Determine soil moisture content
Atterberg Limits	D4318	Determine soil plasticity
Sieve Analysis	D422	Determine soil grain size distribution
Hydrometer Analysis	D422	Determine clay and silt fraction
Specific Gravity	D854	Determine specific gravity
pH	D4972	Determine acidity
Organic Content	D2974	Determine organic content
Permeability	D5084	Determine soil permeability
Consolidation	D2435	Determine soil compressibility
unconsolidated-undrained (UU) triaxial	D2850	Determine undrained shear strength
consolidated-undrained (CIU) triaxial	D4767	Determine drained shear strength

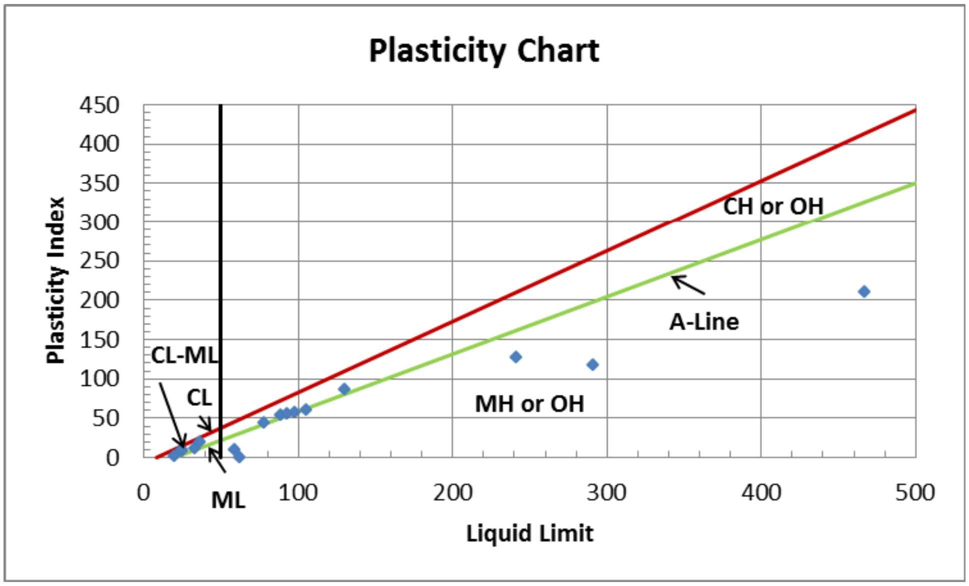
5.2.1. Classification Testing

Results of classification testing are summarized in Table 5-3. Natural Moisture Content results are shown on Test Boring Logs in Appendix B and grain-size distribution graphs, specific gravity, pH, organic content, consolidation, UU triaxial, CIU triaxial test results in Appendix C.

Table 5-3. Laboratory Classification Results							
Sample	**Depth (ft)**	**Description**	**LL%**	**PL%**	**NMC**	**% Clay/Fines**	**USCS**
BD-01R/T-1	24.0-26.0	Elastic SILT with Sand (Peat)	291	173	72.9	16.5/79.1	MH
BD-03/T-1	23.0-25.0	Sandy Elastic SILT (Peat)	105	45	65.1	18.3/66.5	MH
BD-03/S-12	25.0-27.0	Sandy Fat CLAY (Peat)	89	35	73.8	13.8/56.6	CH
BD-04R/T-1	21.0-23.0	Fat CLAY (Peat)	98	41	75.0	17.9/93.2	CH
BD-05/S-5	14.0-16.0	Fat CLAY with Sand (Peat)	78	34	70.9	16.2/82.3	CH
BS-01/S-9	16.0-18.0	Sandy Lean CLAY	25	17	20.4	17.4/64.6	CL
BS-02/T-1	7.0-9.0	Sandy Elastic SILT (Peat)	467	256	375.7	21.5/62.0	MH
BS-03/T-1	8.0-10.0	Elastic SILT (Sludge)	-	-	87.3-124.3	18.0/93.0	MH
BS-04/T-1	23.0-25.0	Fat CLAY (Peat)	93	38	71.9	21.2/87.3	SM
BS-04/S-12	25.0-27.0	Silty SAND	-	-	45.0	-/32.0	SM
BS-06/U-1	2.0-4.0	Elastic SILT (Sludge)	-	-	73.6-84.9	21.0/96.0	MH
BS-08/T-1	15.0-17.0	SILT with Sand	20	18	20.1	7.0/71.1	ML
BS-08/S-10	21.0-23.0	SILT with Sand	-	-	16.0	-/80.0	ML
BS-09/T-1	12.0-14.0	Silty SAND (Peat)	241	113	58.6	7.7/43.1	SM
ID-01/S-3	4.0-6.0	Elastic SILT (Sludge)	59	49	134.3	5.0/96.6	MH
ID-03R/T-1	0.0-2.0	Elastic SILT (Sludge)	-	-	114.5-170.1	26.0/86.7	MH
ID-06/S-2	2.0-4.0	Elastic SILT (Sludge)	62	61	109.2	-/-	MH
ID-06/U-2	14.0-16.0	Fat CLAY (Peat)	130	43	120.9	-/-	CH
IS-02/U-1	19.0-21.0	Sandy Lean CLAY (Peat)	33	22	99.3	-/-	CL
W-05/T-1	15.0-17.0	Sandy Lean CLAY (Peat)	25	15	26.1	-/-	CL
W-08/U-1	14.0-16.0	Lean CLAY (Peat)	36	16	30.4	-/-	CL
COMPOSITE	-	Elastic SILT	NP	64	94.3-115.8	-/-	MH

USCS: Unified Soil Classification System **PL:** Plastic Limit **LL:** Liquid Limit **NMC:** Natural Moisture Content

In general, the liquid and plastic limits of the on-site organic peat material are very high, which is characteristic of these materials. The sludge material generally has lower liquid and plastic limits. There are some peat soils that have lower limits, which may indicate some mixing of the peat and overlying sludge material. The accompanying figure below presents classification results on a plasticity chart.



5.2.2. Permeability Testing

One sample was tested to determine the coefficient of permeability of the sludge using the falling head constant volume method within a triaxial cell. The test was performed on a composite sludge sample taken from several Shelby tubes to determine the flow characteristics of the sludge in a remolded condition. Numerous attempts were made to collect Shelby tube samples within the sludge, however many were unsuccessful due to the low shear strength of the sludge. Many of the Shelby tube samples collected tended to be within the upper portions of the sludge, which tended to be drier and stronger and not necessarily representative of the entire sludge profile. The sample was remolded to a specified density and molding water contents (similar to the undisturbed samples collected), consolidated to 700 psf and then tested to estimate permeability. The pre-test moisture content and dry density were 71.89% and 103.0 pcf, respectively. Table 5-4 presents the results of the permeability testing.

Table 5-4. Permeability Testing Result

Sample	Depth (ft)	Description	Consolidation Pressure (psf)	Permeability (cm/sec)
COMP	-	Elastic SILT (Sludge)	700	$6.43(10)^{-6}$

Note: composite compacted from BS-03/T-1, BS-06/U-1, and ID-03R/T-1

The remolded permeability of the sludge is in the lower range of permeability values measured by previous investigators. The data sheets are presented within Appendix C.

5.2.3. Shear Strength Testing

Shear strength testing was conducted on a total of eleven (11) undisturbed Shelby tube samples and two (2) composited, remolded samples; nine (9) unconsolidated-undrained (UU) compression tests and four (4) consolidated isotropically undrained compression (CIUC) tests. UU testing was conducted on undisturbed soils from borings BD-01R/T-1 (24.0’-26.0’), BD-04R/T-1 (21.0’-23.0’), BS-02/T-1 (7.0’-9.0’), BS-04/T-1 (23.0’-25.0’), BS-08/T-1 (15.0’-17.0’), IS-02/U-1 (19.0-21.0’), W-05/T-1 (15.0’-17.0’), and W-08/U-1 (14.0-16.0’. UU testing was also conducted on a composited, remolded sample of sludge. The CIUC tests were conducted on undisturbed soil from borings BD-03/T-1 (23.0’-25.0’), BS-09/T-1 (12.0’-14.0’), and BS-03R/T-1 (8.0’-10.0’) and a composited, remolded sample of sludge. Appendix E contains photographs of these samples before, during, and after testing.

UU compression tests are performed within a triaxial cell with the drainage lines closed. A minimal cell pressure is applied followed by axial loading, therefore, only a small amount of consolidation is allowed to occur. Measured soil parameters depend heavily upon degree of saturation. The confining pressure is atmospheric (or zero gauge pressure). CIUC compression tests involve the initial consolidation of the sample under some designated confining cell pressure with the drainage lines open. Under isotropic conditions, the confining cell pressure is equal in all directions. After consolidation is complete, the drainage lines are closed and the sample is compressed.

UU and CIUC samples contained in shelby tubes were extruded, trimmed and set-up at the specified confining pressures of 2,000 psf for BD-01R/T-1 (24.0’-26.0’), 2,500 psf for BD-04R/T-1 (21.0’-23.0’), 900 psf for BS-02/T-1 (7.0’-9.0’), 2,800 psf for BS-04/T-1 (23.0’-25.0’), 1,700 psf for BS-08/T-1 (15.0’-17.0’), 2,300 psf for IS-02/U-1 (19.0-21.0’), 1,600 psf for W-05/T-1 (15.0’-17.0’), and 1,700 psf for W-08/U-1 (14.0-16.0’. The confining pressures were chosen to simulate the existing loading conditions of the samples when they were collected. The confining pressure for the composited, remolded sludge was 1,000 psf. The CIUC samples were also contained within Shelby tubes and were extruded, trimmed and set-up within triaxial cells for testing. The samples were back-pressured up to 100 psi to improve the degree of saturation. They were then consolidated to specified effective consolidation pressures of 500, 2,000 and 5,000 psf for BD-03/T-1 (23.0’-25.0’) and BS-09/T-1 (12.0’-14.0’), and 200, 2,000 and 5,000 psf for BS-03R/T-1 (8.0’-10.0’) and then sheared under undrained conditions. The effective consolidation pressures for the composited, remolded sludge sample were also 200, 2,000 and 5,000 psf. Table 5-5 presents the results.

Table 5-5. Triaxial Compression Test Results					
Boring/ Sample	Depth (ft)	Description	Undrained Shear Strength	Effective Stress Parameters	
			S_u (psf)	ϕ'-angle	c' (psf)
BD-01R/T-1	24.0-26.0	Elastic SILT with Sand (MH)	665	-	-
BD-03/T-1	23.0-25.0	Sandy Elastic SILT (MH)	-	27.4	450
BD-04R/T-1	21.0-23.0	Fat CLAY (CH)	390	-	-
BS-02/T-1	7.0-9.0	Sandy Elastic SILT (MH)	360	-	-
BS-03/T-1	8.0-10.0	Elastic SILT (MH)	-	63.4	0
BS-04/T-1	23.0-25.0	Fat CLAY (CH)	550	-	-
BS-08/T-1	15.0-17.0	SILT with Sand (MH)	305	-	-
BS-09/T-1	12.0-14.0	Silty SAND (SM)	-	33.6	550
IS-02/U-1	19.0-21.0	Sandy Lean CLAY (CL)	155	-	-
W-05/T-1	15.0-17.0	Sandy Lean CLAY (CL)	175	-	-
W-08/U-1	14.0-16.0	Lean CLAY (CL)	305	-	-
Composite	-	Elastic SILT (MH)	30	44.0	0
S_u: Undrained Shear Strength; **ϕ, ϕ':** Angle of Internal Friction (total and effective); **c, c':** cohesion (total and effective)					

After testing of the samples, the failure planes were analyzed and photographed (see Appendix E). In overconsolidated clays, the clays tend to expand or dilate when sheared, creating negative pore pressures, while normally consolidated clays contract when sheared, creating positive pore pressures as pore water is squeezed out. As shown on the CIUC data sheets within appendix B, excess pore pressures are generally positive for samples BD-03 and BS-09. For sludge sample BS-03, the excess pore pressures are initially positive but become slightly negative as strain continues. Since this sample was collected relatively close to ground surface, this may indicated that the sludge has dried out to a degree, therefore appearing to be slightly overconsolidated.

While in the field, the Baker field engineer conducted pocket penetrometer testing of the SPT samples. The pocket penetrometer is a hand-held device with a calibrated spring. The device is pushed into the sample until the clay is penetrated a certain distance. The resulting value of unconfined compressive strength is approximately equal to twice the undrained shear strength, as illustrated below in Table 5-6.

Table 5-6. Pocket Penetrometer Results

Boring	Depth (ft)	Description	Unconfined Compressive Strength (tsf)	Undrained Shear Strength (psf)
BD-01R	32	Clayey SAND	1.25	1.250
	42	Clayey SAND	4.0	4,000
BD-03	19	Sandy Elastic SILT (peat)	0	0
	21	Sandy Elastic SILT (peat)	0	0
BD-04R	0	Clayey SILT (fill)	4.5	4,500
	2	Elastic SILT (sludge)	1	1,000
	23	Fat CLAY (peat)	0	0
BD-05	9	Fat CLAY with Sand (peat)	0	0
	12	Fat CLAY with Sand (peat)	0	0
	36	Silty SAND	0.5	500
BS-01	10	Sandy Lean CLAY	0.5	500
	12	Sandy Lean CLAY	0.5	500
	14	Sandy Lean CLAY	1.0	1,000
	16	Sandy Lean CLAY	2.0	2,000
	18	Sandy Lean CLAY	1.0	1,000
	23	Sandy Lean CLAY	0.25	250
	24	Sandy Lean CLAY	1.0	1,000
	25	Sandy Lean CLAY	1.5	1,500
BS-02	4	Elastic SILT (sludge)	0	0
BS-05	16	Silty SAND	<0.01	<10
BS-06	10	Fat CLAY (peat)	0	0
	16	Fat CLAY (peat)	0	0
	18	Fat CLAY (peat)	<0.05	<50
BS-08	6	Organic CLAY (peat)	0.25	250
	8	Organic CLAY (peat)	0	0
	10	Organic CLAY (peat)	0	0
	12	Organic CLAY (peat)	0	0
	13	SILT with Sand	0	0
ID-01	10	Elastic SILT (sludge)	<0.1	<100
	12	Elastic SILT (sludge)	<0.1	<100
	18	Fat CLAY (peat)	<0.1	<100
	24	Fat CLAY (peat)	<0.1	<100
ID-01	35	Clayey SILT	1.0	1,000
ID-02	14	Elastic SILT (sludge)	0	0
	16	Sandy Silty CLAY (peat)	0	0
	18	Sandy Silty CLAY (peat)	0	0

Table 5-6. Pocket Penetrometer Results

Boring	Depth (ft)	Description	Unconfined Compressive Strength (tsf)	Undrained Shear Strength (psf)
	23	Sandy Silty CLAY (peat)	0	0
	25	Sandy Silty CLAY (peat)	0	0
ID-06	16	Fat CLAY (peat)	0	0
	18	Fat CLAY (peat)	0.15	150
	19.3	Silty SAND	2.5	2,500
IS-01	12	Lean CLAY (peat)	0	0
	17	Lean CLAY (peat)	0	0
	21	Lean CLAY (peat)	0	0
	23	Lean CLAY (peat)	0	0
IS-02	14	Sandy Lean CLAY (peat)	0	0
	16	Sandy Lean CLAY (peat)	0	0
W-01	8	Fat CLAY (peat)	0	0
	14	Fat CLAY (peat)	0	0
	27	Silty GRAVEL with Sand	2.0	2,000
W-03	6	Fat CLAY (peat)	0	0
	8	Fat CLAY (peat)	0	0
	10	Fat CLAY (peat)	0	0
	17.8	Silty GRAVEL	2	2,000
W-04	8	Elastic SILT (peat)	0	0
	16	Elastic SILT (peat)	0	0
W-05	4	Sandy Lean CLAY (peat)	0	0
	6	Sandy Lean CLAY (peat)	0	0
	8	Sandy Lean CLAY (peat)	0	0
	10	Sandy Lean CLAY (peat)	0	0
	12	Sandy Lean CLAY (peat)	0	0
W-07R	15	Sandy SILT	1.25	1,250
	20	Sandy SILT	1.5	1,500
W-08	8	Lean CLAY (peat)	0	0
	10	Lean CLAY (peat)	0	0
	16	SILT	2.0	2,000

5.2.4. Consolidation Testing

Eight (8) one-dimensional consolidation tests were performed on undisturbed Shelby tube samples from borings BD-04R/T-1 (21.0’-23.0’), BS-02/T-1 (7.0’-9.0’), BS-03/T-1 (8.0’-10.0’), BS-06/U-1 (2.0’-4.0’), BS-09/T-1 (12.0’-14.0’), ID-03R/T-1 (0.0-2.0’), ID-06/U-2 (14.0’-16.0’), and IS-02/U-1 (19.0’-21.0’).

One (1) consolidation test was performed on a composted, remolded sample of sludge. Consolidation tests are performed on saturated samples placed within a confining metal fixed-ring or floating ring apparatus. As load is applied to the sample, water flows from the sample and the sample volume subsequently reduces. The samples were extruded, trimmed and set-up within Antius® consolidometers and were then loaded up to 1, 2, 4, or 6 tons per square foot (tsf)) before initial unloading and then reloaded up to 2, 8, 12, or 16 tsf to better define the virgin slope and pre-consolidation characteristics. Table 5-7 presents the results.

Table 5-7. Consolidation Test Results

Boring/ Sample	Depth (ft)	Description	C_c	C_r	e_o	P_c (psf)	P_o (psf)	C_v (ft^2/day) @ 2 tsf	OCR
BD-04R/T-1	21.0-23.0	Fat CLAY	0.83	0.13	2.000	1,440	1,450	0.07	1.0
BS-02/T-1	7.0-9.0	Sandy Elastic SILT	5.64	0.64	8.150	920	380	0.01	2.4
BS-03/T-1	8.0-10.0	Elastic SILT (Sludge)	1.29	0.03	3.866	4,800	538	4.93	8.9
BS-06/U-1	2.0-4.0	Elastic SILT (Sludge)	0.77	0.03	2.790	6,400	336	1.37	19.1
BS-09/T-1	12.0-14.0	Silty SAND	0.48	0.06	1.279	1,660	870	2.07	1.9
ID-03R/T-1	0.0-2.0	Elastic SILT (Sludge)	1.10	0.05	4.289	800	104	0.61	7.7
ID-06/U-2	14.0-16.0	Fat CLAY	0.79	0.13	2.466	980	600	0.11	1.6
IS-02/U-1	19.0-21.0	Sandy Lean CLAY	0.41	0.04	0.904	2,160	1,060	0.02	2.0
Composite	-	SILT (Sludge)	0.60	0.06	3.704	-	-	3.00	-

C_c: Compression Index **C_r**: Recompression Index **C_s**: Swell Index **P_c**: Preconsolidation Pressure
P_o: Current Overburden Pressure **C_v**: Coefficient of Consolidation **OCR**: Overconsolidation Ratio

5.2.5. Miscellaneous Testing

Specific Gravity (ASTM D854). Nine (9) samples were analyzed to determine specific gravity, including BD-03R/T-1 (24.0'-26.0'), BD-03/T-1 (23.0'-25.0'), BS-02/T-1 (7.0'-9.0'), BS-03/T-1 (8.0'-10.0'), BS-06/U-1 (2.0'-4.0'), BS-09/T-1 (12.0'-14.0'), ID-03R/T-1 (0.0-2.0'), ID-06/U-2 (14.0'-16.0'), and IS-02/U-1 (19.0'-21.0'). Specific gravity is useful in determining the void ratio of a soil and to determine the density of a soil. The results are tabulated below in Table 5-8.

Table 5-8. Specific Gravity Testing Results

Boring/Sample	Depth (m)	Description	Specific Gravity
BD-01R/T-1	24.0-26.0	Elastic SILT with Sand	2.36

Table 5-8. Specific Gravity Testing Results			
Boring/Sample	**Depth (m)**	**Description**	**Specific Gravity**
BD-03/T-1	23.0-25.0	Sandy Elastic SILT	2.53
BS-02/T-1	7.0-9.0	Sandy Elastic SILT	2.14
BS-03/T-1	8.0-10.0	Elastic SILT (Sludge)	3.17
BS-06/U-1	2.0-4.0	Elastic SILT (Sludge)	3.10
BS-09/T-1	12.0-14.0	Silty SAND (SM)	2.52
ID-03R/T-1	0.0-2.0	Elastic SILT (Sludge)	2.76
ID-06/U-2	14.0-16.0	Fat CLAY (CH)	2.58
IS-02/U-1	19.0-21.0	Sandy Lean CLAY (CL)	2.63

The specific gravity of the sludges on site varies between 2.76 and 3.17, substantially higher than typically encountered soils.

Organic Content (ASTM D2974). Four (4) samples were analyzed to determine organic content, including BD-01R/T-1 (24.0'-26.0'), BD-05/S-5 (14.0'-16.0'), BS-02/T-1 (7.0'-9.0'), and BS-09/T-1 (12.0'-14.0'). The purpose of this testing was to determine the potential for secondary long-term creep settlement due to compression of organic materials. The results were fairly low as shown in Table 5-9.

Table 5-9. Organic Content Testing Results			
Boring/Sample	**Depth (m)**	**Description**	**Organic Content (%)**
BD-01R/T-1	24.0-26.0	Elastic SILT with Sand (MH)	14.8
BD-05/S-5	14.0-16.0	Fat CLAY with Sand (CH)	5.7
BS-02/T-1	7.0-9.0	Sandy Elastic SILT (MH)	19.3
BS-09/T-1	12.0-14.0	Silty SAND (SM)	2.9

Based upon the organic content of these samples, BD-01R/T-1 and BS-02/T-1 would not necessarily be classified as peat, but do have moderately high organic contents. The other two samples are not organic soils.

pH (ASTM D4972). Four (4) samples were analyzed to determine pH, including BD-01R/T-1 (24.0'-

26.0’), BD-05/S-5 (14.0’-16.0’), BS-02/T-1 (7.0’-9.0’), and BS-09/T-1 (12.0’-14.0’). The purpose of this testing was to determine whether these samples are organic materials. The results are illustrated in Table 5-10.

Table 5-10. pH Testing Results

Boring/Sample	Depth (m)	Description	pH
BD-01R/T-1	24.0-26.0	Elastic SILT with Sand (MH)	6.9
BD-05/S-5	14.0-16.0	Fat CLAY with Sand (CH)	6.6
BS-02/T-1	7.0-9.0	Sandy Elastic SILT (MH)	6.7
BS-09/T-1	12.0-14.0	Silty SAND (SM)	6.5

Based upon the pH of these samples, they do not appear to be peat.

8.0 GEOTECHNICAL RECOMMENDATIONS

Both the settlement and the stability analysis indicate that the proposed grading plan can be safely achieved as long as construction is performed in stages with waiting periods between stages. It is further understood that the completion of the fill will take approximately 5 years; therefore this should not be an issue. Based on the current available data, 90% consolidation of the cohesive soil underneath the sludge will take two to three years after the placement of fill. For a sludge layer 13 feet thick, the estimated time required to reach 90% of consolidation is approximately 48 days.

The following construction sequence is recommended:

1. Prepare the Subgrade

    - Only cut trees and stumps flush with ground surface.
    - Do not remove or disturb root mat or meadow mat.
    - Leave small vegetative cover, such as grass and reeds, in place. The vegetation will help to carry the construction equipment load during the site preparation and placing the first several feet of the fill.

    Prior to receiving materials without permeability requirements, a minimum of three (3) feet of "fair drainage material" shall be placed on top of the current impoundments. The "fair drainage material" is defined as soil materials with laboratory tested permeability greater than 5.0E-05 cm/s. The laboratory permeability test procedures should conform to ASTM D2434 or ASTM D5856. The sample for the permeability test should have a minimum of 90% maximum dry density as determined by ASTM D698 (Standard Compaction Effort). The following procedure should be followed:

    a. End-dump the fill materials into the impoundment from the adjacent access road. The first lift of the fill should consist of "fair drainage material".

        - Use appropriate trucks and equipment compatible with constructability design.
        - End-dump on the previously placed fill;
        - Limit height of dumped piles, e.g., to less than four (4) ft above the adjacent sludge or five (5) feet above the previous fill, to avoid a local bearing failure. Spread piles immediately to avoid local depressions.
        - Use lightweight dozers and/or front-end loaders to spread the fill.

    b. Traffic on the first lift should be limited.

- Construction vehicles should be limited in size and weight to limit initial lift rutting to 3 inches. If rut depths exceed 3 inches, decrease the construction vehicle size and/or weight.

c. The first lift should be compacted only by tracking in place with low-ground-pressure bulldozers or end-loaders.

d. Once the cap is at least 2 ft above the original ground, subsequent lifts can be compacted with a smooth drum vibratory roller or other suitable compactor. If localized liquefied conditions occur, the vibrator should be turned off and the weight of the drum alone should be used for compaction.

e. Generally, the above procedure applies to Impoundments 1, 4, 5 and 6. Minor adjustment of the thickness of the first lift and the construction sequence might be required during construction.

2. After the “fair drainage material” (first lift) is placed, the geotechnical instrumentation needs to be installed including settlement plates, vibrating wire piezometers and slope inclinometers. Special Provisions for instrumentation will be developed during final design when a draft fill plan is available. The following are the brief summary:

   a. The settlement plates should be placed at the top of the first lift. The objective of the settlement plate installation is to control the construction sequence.
   b. For the piezometers located near the river, filters shall be installed in both organic cohesive soil as well as the sludge. At other locations, the piezometer filters are only required to be installed within the sludge. The objective of installing piezometers is to monitor the dissipation of excess pore pressures generated during construction. The shear strength of the soil will increase after dissipation.
   c. The slope inclinometers should be installed along the edge of the proposed fill to ensure that slope stability is maintained.
   d. Additional geotechnical in-situ tests such as CPTu and field vane shear testing will be required during construction.

3. End of Stage One construction. The starting time for the next stage of construction will be based on the instrumentation monitoring results. Generally, the criterion for the next construction stage is when

the soft material reaches a minimum of 90% of primary consolidation. The estimated waiting period is approximately 38 days.

4. At areas near the river (Impoundments 4 and 6), the fill placement might also be controlled by dissipation of excess pore pressure within the in-situ cohesive soils underlying the sludge. A longer waiting period time might be required. For construction activities in these areas, more frequent instrumentation monitoring will be required to ensure the rate of fill placement is slow enough to allow the dissipation of excess pore pressure.

5. Place the embankment and surcharge fills for each construction stage. The thickness of each construction should not exceed 5 ft. The fill material needs to be roller compacted.

6. To facilitate the dissipation of the excess pore water pressure in the in-situ cohesive soil underlying the sludge, place all fill material in a pattern with relatively the same elevation.

7. Due to the relatively high shear strength of surface material at Impoundments 2 and 3, fill material should be no more than ten feet in height above the adjacent ground. At all other areas, piles should be no more than six (6) feet in height. If higher stockpile heights are desired, it is recommended that the stockpiles be placed in a staged manner, thereby allowing the underlying soils to consolidate. Once the stockpile has been reached the desired height, further staged placement should not be required. The consolidation of the underlying materials should also be monitored with field shear strength testing completed prior to building the stockpiles to greater heights.

10.0 BASIS OF RECOMMENDATIONS

This report has been prepared to aid in the evaluation for the proposed construction described in this report. Adequate recommendations have been provided to serve as a basis for design and preparation of plans and specifications. The opinions, conclusions, and recommendations contained in this report are based upon our professional judgment and generally accepted principles of geotechnical engineering. Inherent to these are the assumptions that the earthwork and foundation construction should be monitored and tested by an engineering technician acting under the guidance of a licensed geotechnical engineer.

These analyses and recommendations are, of necessity, based on the information available at the time of the actual writing of the report and on the site conditions, surface and subsurface, that existed at the time the exploratory borings were drilled. Further assumption has been made that the limited exploratory borings, in relation both to lateral extent of the site and to depth, are representative of conditions across the site.

The nature and extent of variations between borings may not become evident until construction. If variations from the anticipated conditions are encountered, it may be necessary to revise the recommendations in this report. We cannot accept the responsibility for designs based on recommendations in this report unless we are engaged to make site visits during construction to: a) check that the subsurface conditions exposed during construction are in general conformance with our design assumptions and b) ascertain that, in general, the work is being performed in compliance with the contract documents.

Our professional services have been performed in accordance with generally accepted engineering principles and practices; no other warranty, expressed or implied, is made. Baker assumes no responsibility for interpretations made by others on the work performed by Baker.

We recommend that this report be made available in its entirety to contractors for informational purposes only. The boring logs and laboratory test data contained in this report represent an integral part of this report and incorrect interpretation of the data may occur if the attachments are separated from the text.